C&EE 141

Tension Members

Tension Members

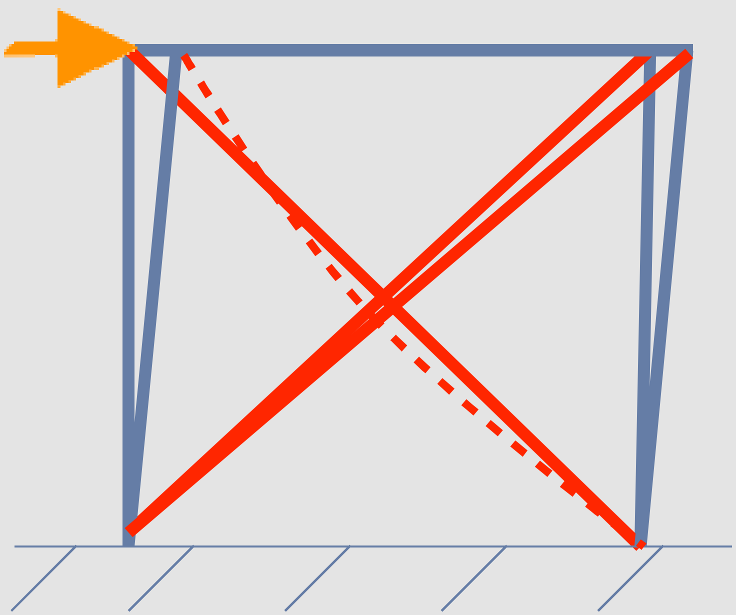
- Definition
  - Structural elements subjected to axial forces that cause elongation
- Applications
  - Hanging supports
  - Chords of trusses that are in tension
  - Tension only rods
  - Braces
- Be Careful
  - Often braces resist both tension and compression forces

Hangers

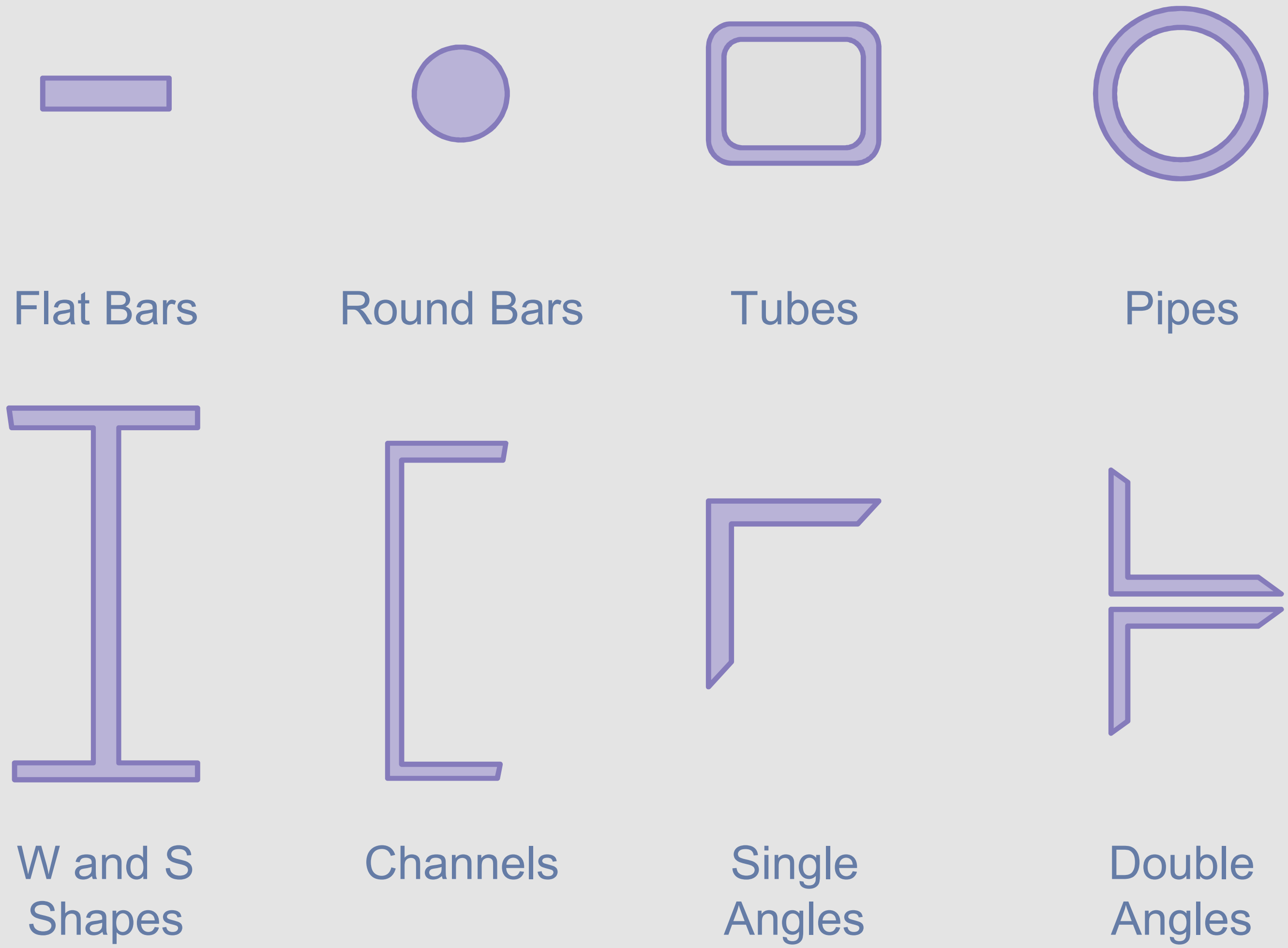
Trusses

- Members in Compression
- Members in Tension

Bracing for Lateral Forces



Any X-Section Can Be Used



Stress in Axially-Loaded Tension Member

$$f = P / A$$

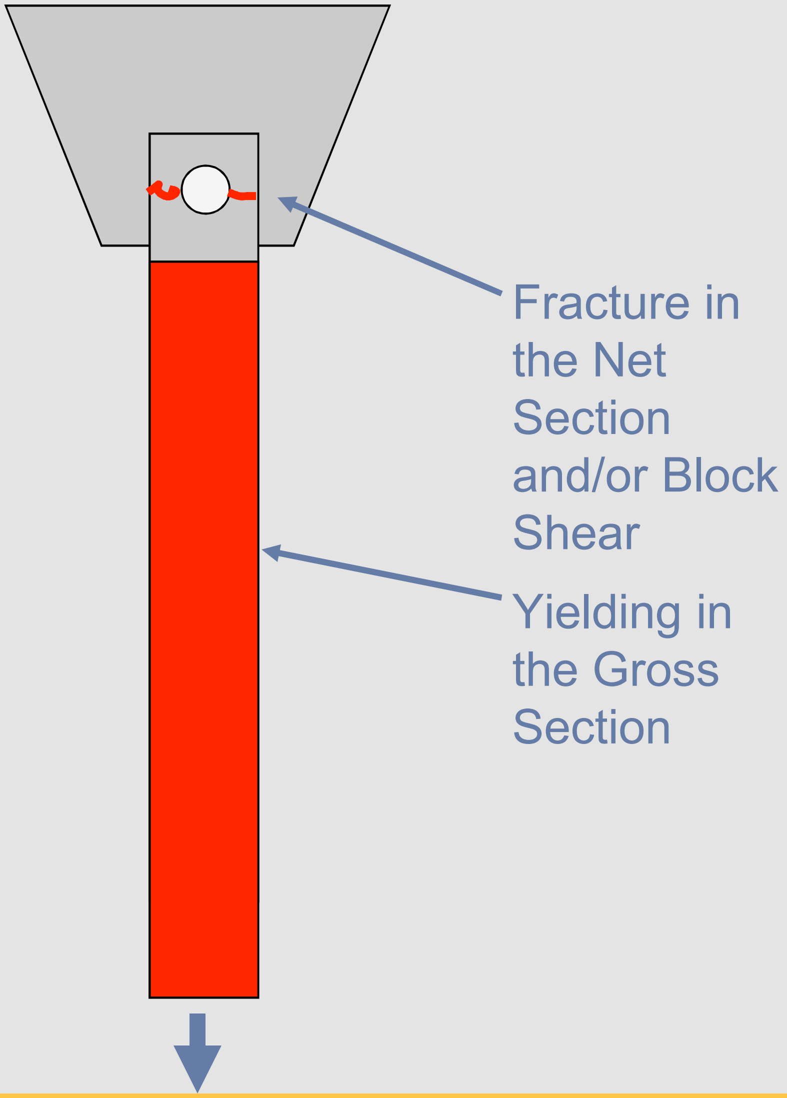
Stress = Load Divided By Area

Simple Example

- Select a member with sufficient area to resist the load
  - $P = 100k$
  - $f = 25$ ksi
  - $A = ?$ in^2

Limit States for Tension Behavior

1. Yielding of the gross section
   - intended to prevent excessive elongation of the member
2. Fracture of the net section
   - e.g. when there are bolt holes present
3. Serviceability
4. Block Shear



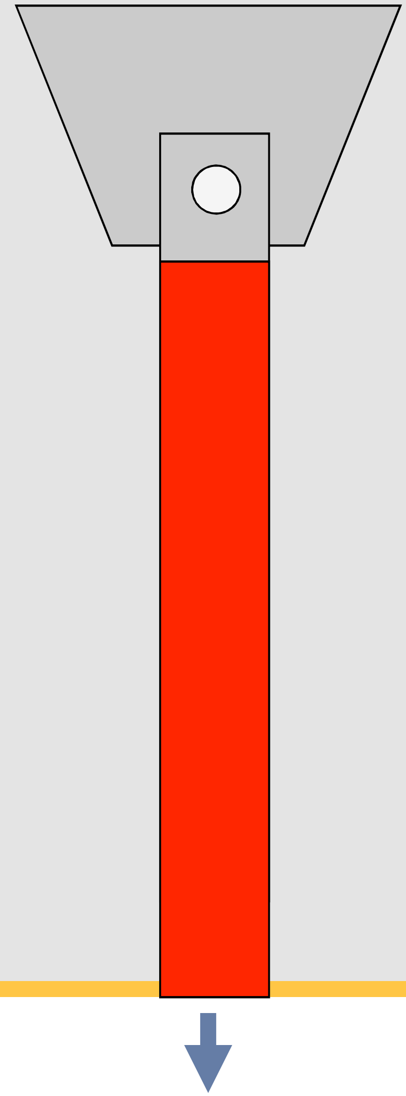
Limit States on Tensile Strength

Prevent Yielding of the Gross X-Section

$$P_n = F_y A_g$$

$$A_g = \text{Gross Area}$$

- Design Capacity = $\Phi_t P_n$ where $\Phi_t = 0.9$
- Refer to Spec Section D2
- F_y per AISC Table 2-4 or 2-5



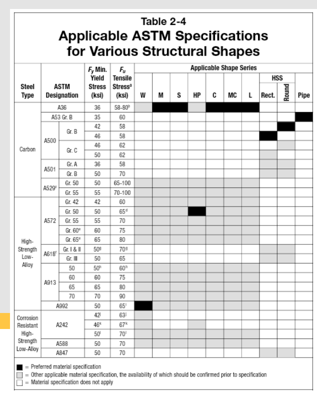


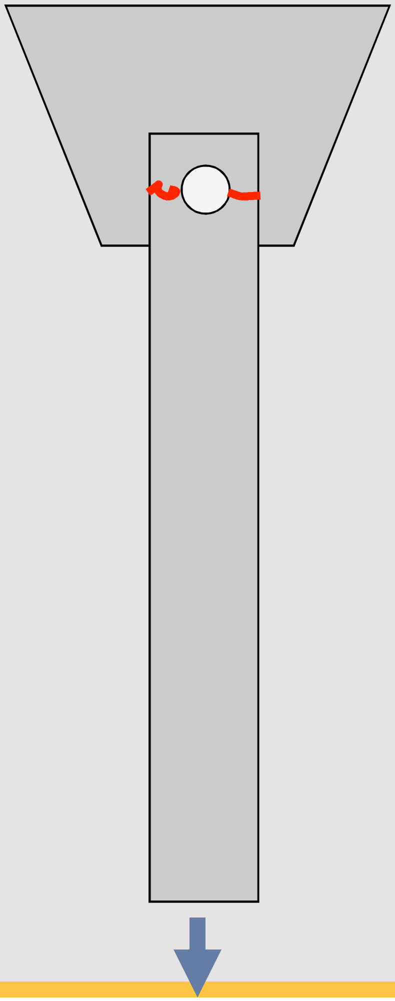
Table 2-4
Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series									
					W	M	S	HP	C	MC	L	HSS Rect.	HSS Round	Pipe
Carbon	A36		36	58-80[b]	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▒	
			50	62								▒		
	A501	Gr. A	36	58								▒	▒	
		Gr. B	50	70								▒	▒	
	A529[c]	Gr. 50	50	65-100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70-100	▒	▒	▒	▒	▒	▒	▒			
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65[d]	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60[e]	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65[e]	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618[f]	Gr. I & II	50[g]	70[g]								▒	▒	
		Gr. III	50	65								▒	▒	
	A913	50	50[h]	60[h]	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50	65[i]	■	▒	▒	▒	▒	▒	▒			
Corrosion Resistant High-Strength Low-Alloy	A242		42[j]	63[j]	▒									
			46[k]	67[k]	▒			▒						
			50[l]	70[l]	▒	▒	▒	▒	▒	▒	▒			
	A588		50	70	▒	▒	▒	▒	▒	▒	▒			
	A847		50	70								▒	▒	

■ = Preferred material specification
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification
□ = Material specification does not apply

Limit States on Tensile Strength

Prevent Rupture of the Net Section



$$P_n = F_u A_e$$

$$A_e = \text{Effective Net Area}$$

- Design Capacity = $\Phi_t P_n$ where $\Phi_t = 0.75$
- Refer to Spec Section D2
- F_u per AISC Table 2-4 or 2-5

Area Determination

- Gross Area (Spec B4.3a)
  - Total cross-sectional area
- Net Area (Spec B4.3b)
  - Reduced cross-sectional area because of bolts or other holes
- Effective Net Area (Spec D3)
  - Reduced cross-sectional area because of *shear lag*

Net Area

- When a connection involves bolts, holes are required
- Therefore, the member x-sectional area is reduced
- In turn, the tensile capacity may be reduced

Hole Types

- Standard Sized Holes
  - Punched
    - 1/16 inch oversized (but assume 1/8…more on that later)
    - <u>Most common process</u>
    - Should be assumed unless otherwise specified
  - Sub-Punched and Reamed
    - Not oversized
    - Expensive process, used only when tight fit-up required
  - Drilled
    - 1/32 inch oversized
    - Done for very thick plates
- Also: Oversized (OS), Short-Slotted (SSL), and Long-Slotted (LSL) Holes

Drilled Holes

Punched Holes

Nominal Hole Dimensions

Spec J3.2

TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter, in.	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1 1/4	1 1/16 × 1 5/16	1 1/16 × 2½
≥ 1 1/8	d + 1/16	d + 5/16	(d + 1/16) × (d + 3/8)	(d + 1/16) × (2.5 × d)

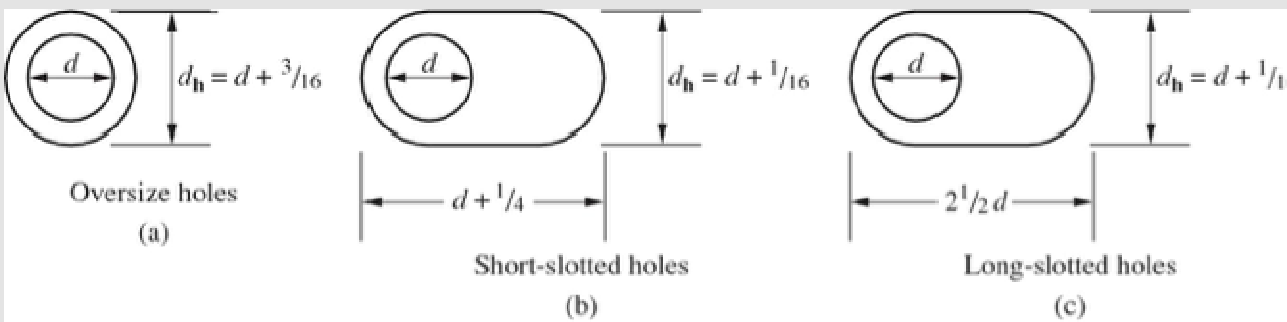


Figure 4.14

Hole Punching

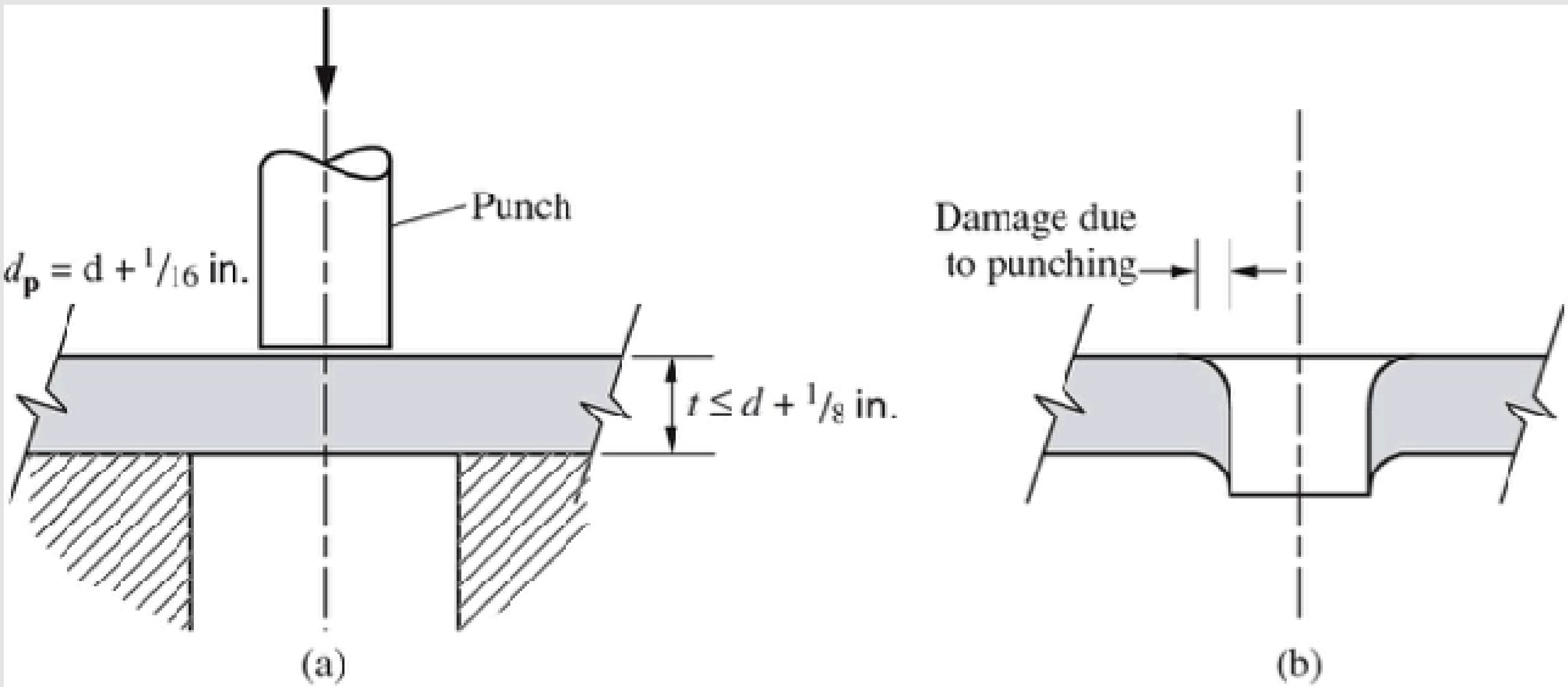
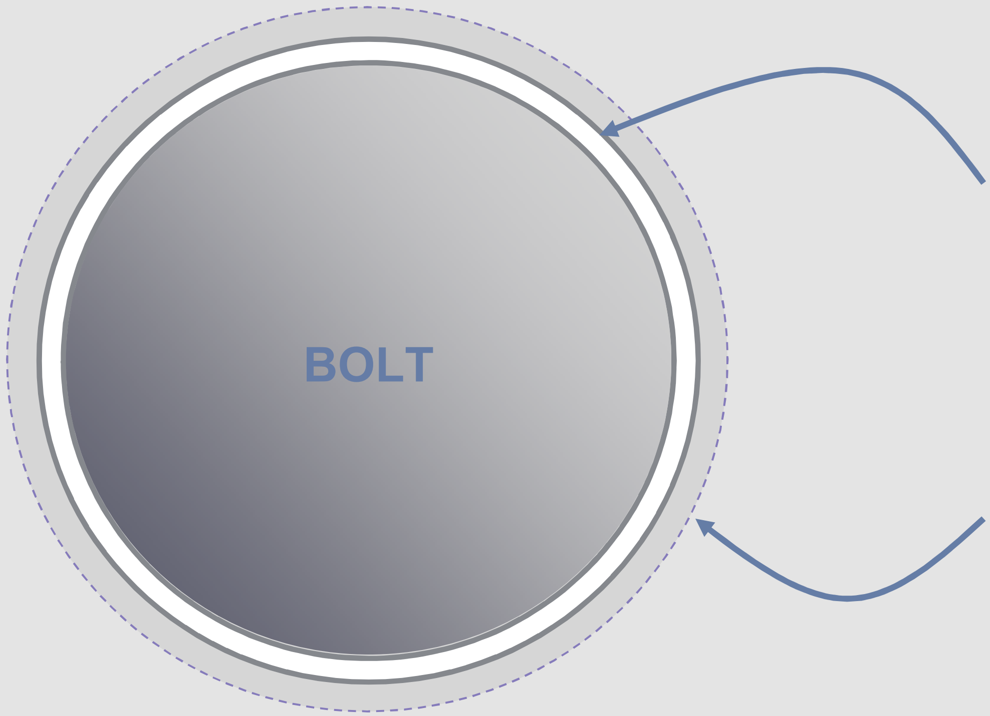


Figure 4.12

Effective Width of a Standard Bolt Hole



Add 1/16 inch to bolt diameter for nominal hole diameter (Spec Table J3.3) (empty space)

Add 1/16 inch to nominal hole diameter for effective hole diameter, due to damage from process to create hole (Spec B4.3b) (damaged base material)

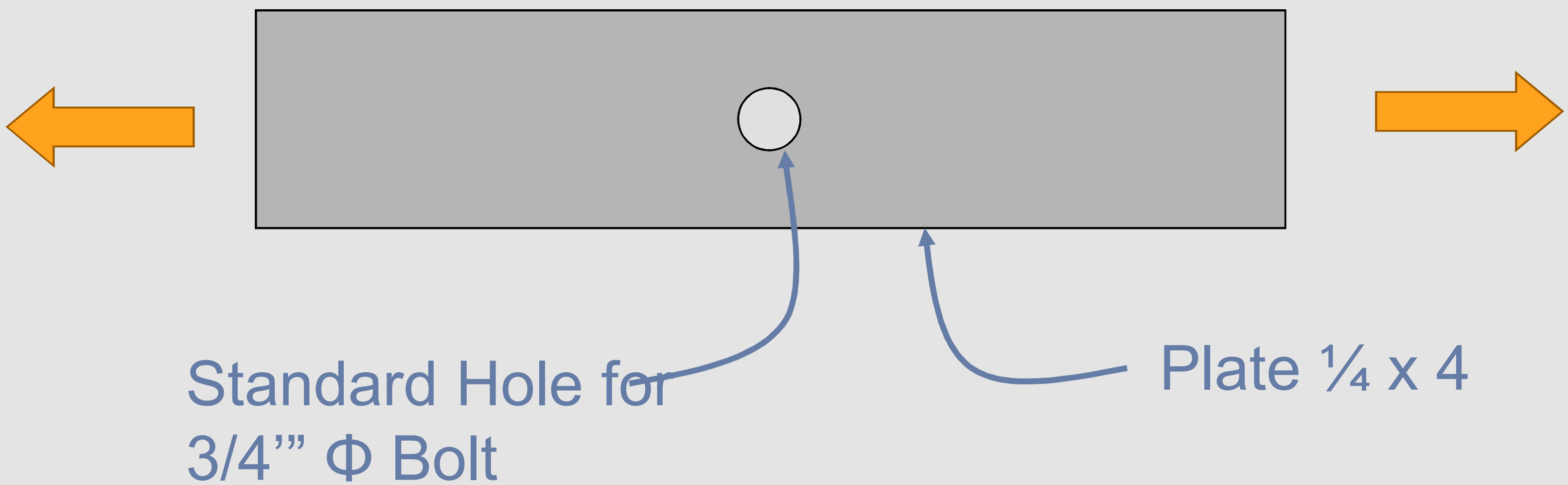
Effective width of bolt hole

= the bolt diameter + 1/16” (oversize) + 1/16” (damage)

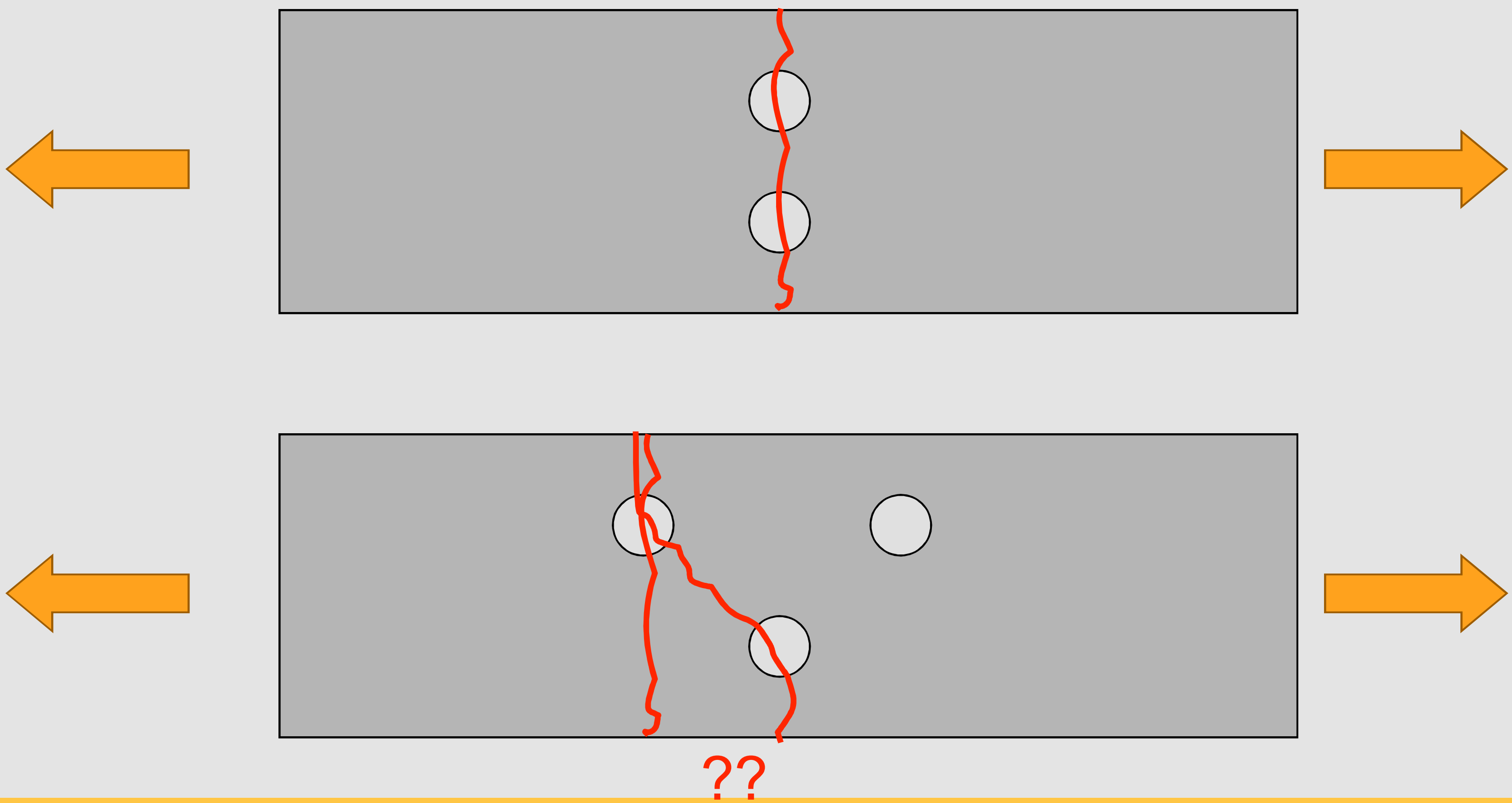
= the bolt diameter + 1/8”

Simple Example

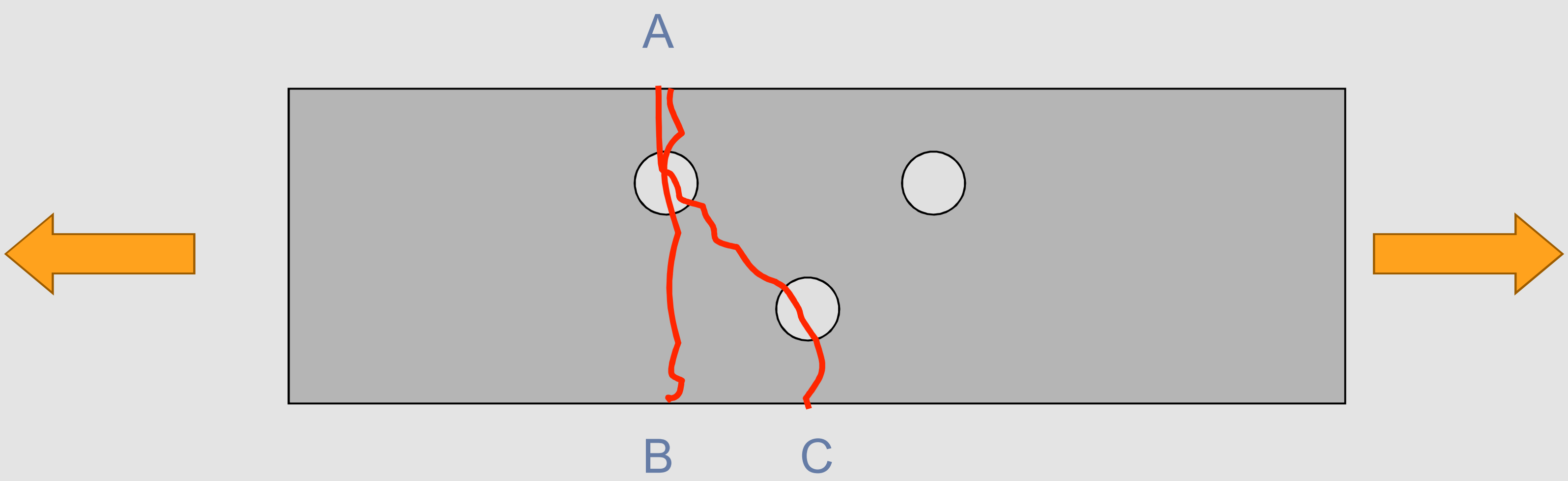
What is the net area for the tension member below?



Effect of Staggered Holes on Net Area



Staggered Holes

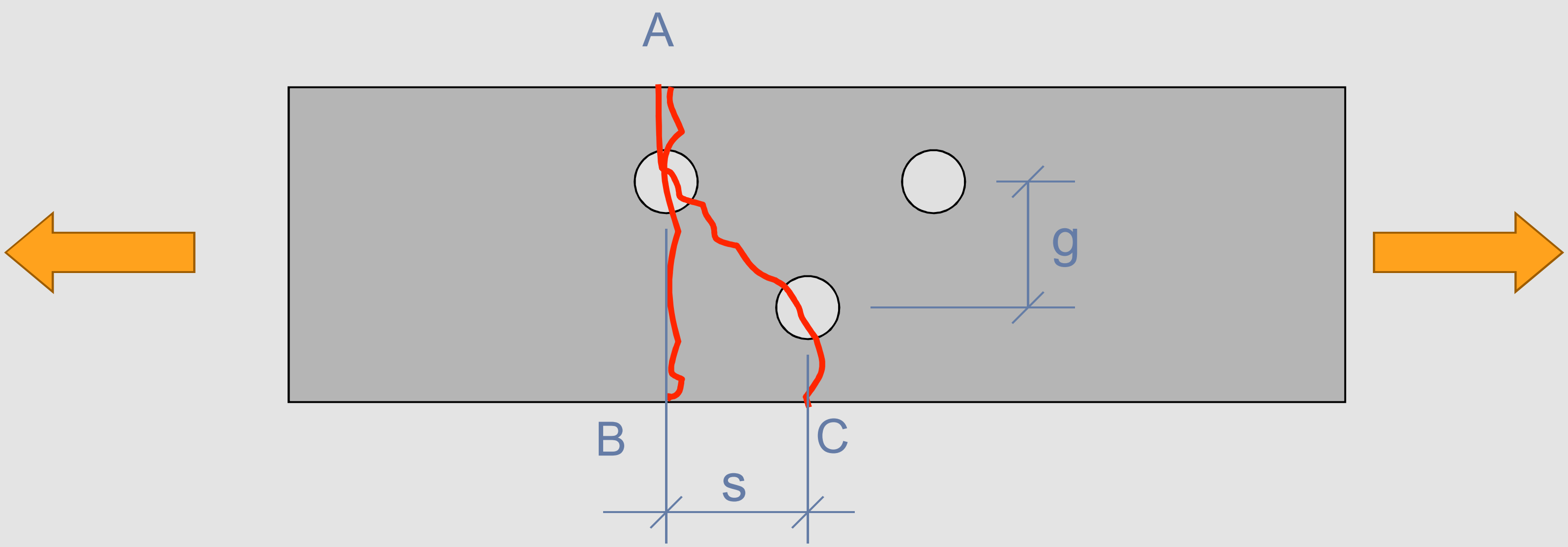


More than one controlling failure line may exist. The controlling failure line is that which gives the minimum net area.

A-B may govern (One Hole), or A-C may govern (Two Holes). Both must be checked.

Accurate checking of strength along A-C is complex. So the Spec provides a simplified empirical procedure (Cochrane - 1922).

Stagger



Length Correction for Stagger = $s^2/4g$

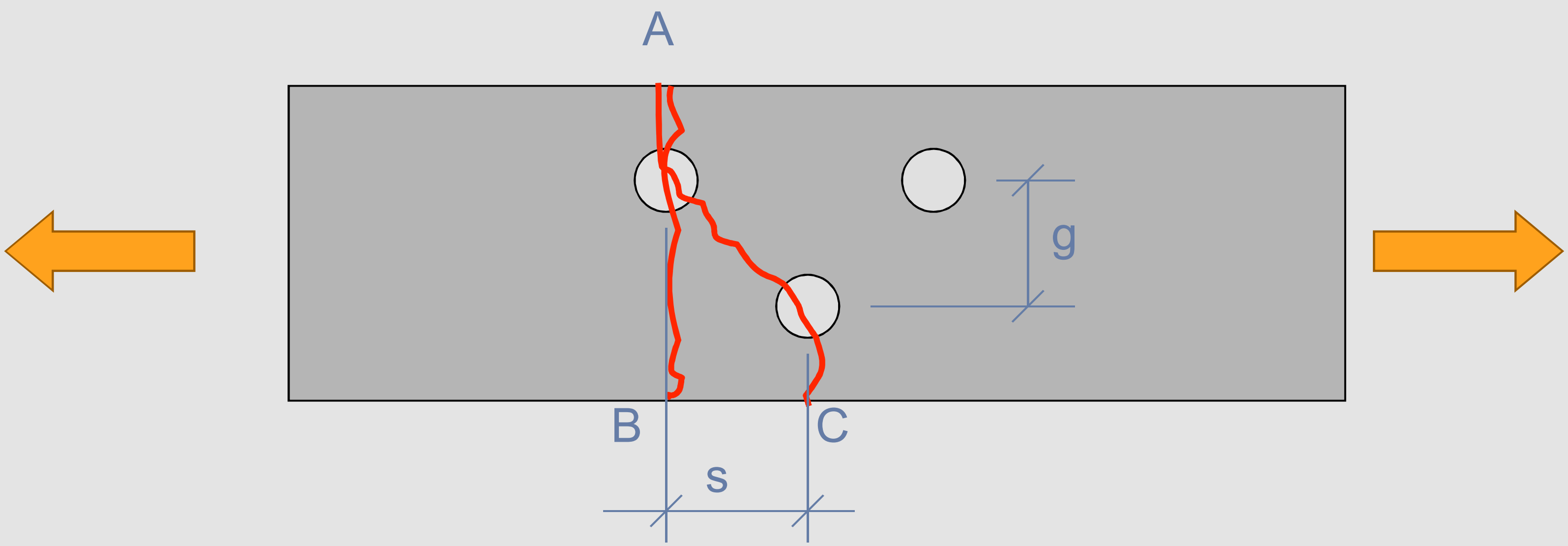
s = spacing or pitch (c-to-c)

g = gage (c-to-c)

Add length correction to net width of part

Spec B4.3b

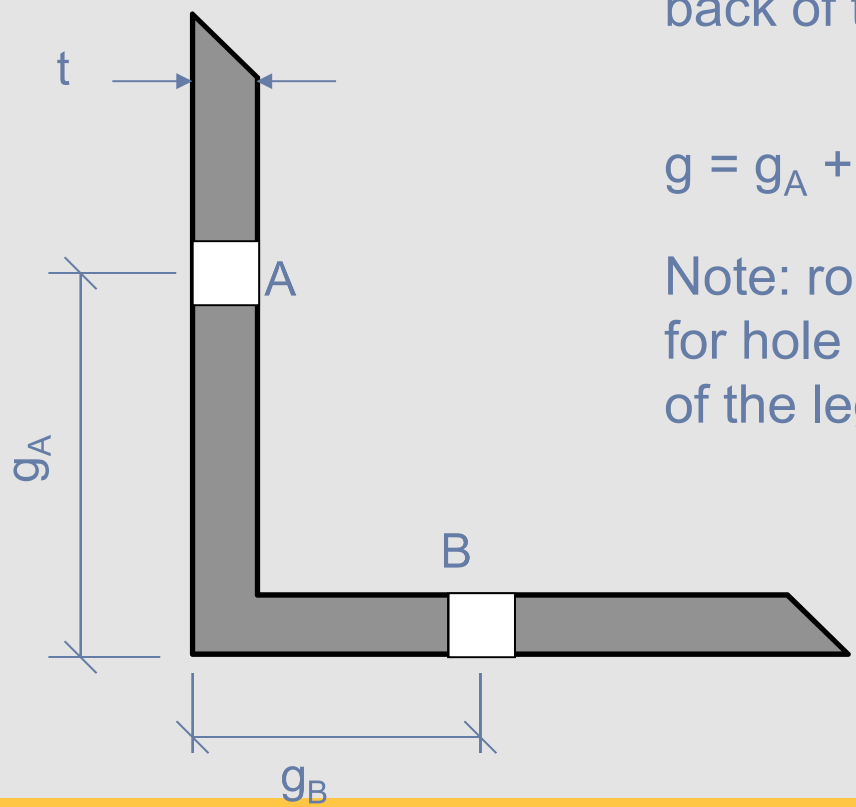
Stagger



Net Length A-B = Length A-B – (Hole Width + 1/8)

Net Length A-C = Length A-B – 2 x (Hole Width + 1/8) + $s^2/4g$

Angles and Net Area



The g in $s^2/4g$ is obtained by summing the gage from the centers of the holes to the back of the angle, less the angle thickness.

$$g = g_A + g_B - t$$

Note: rolled angles have standard gages for hole locations depending on the length of the leg (Table 1-7A)

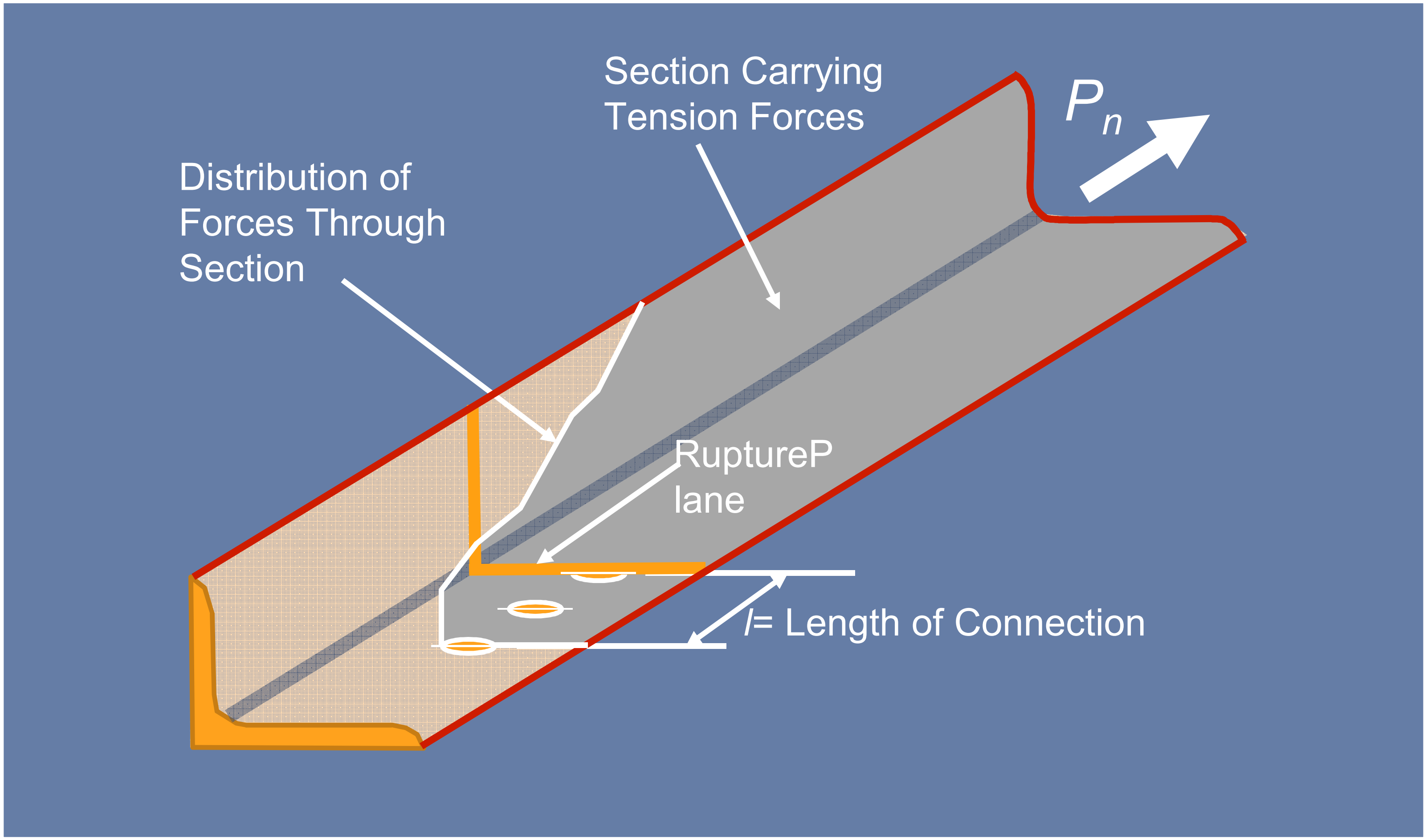
Effective Net Area

- Accounts for the efficiency of the connection
- "Shear Lag" effect:
  - Occurs when not all elements in a cross-section are part of connection
  - Non-uniform stress distribution between the connected and un-connected elements occurs
  - A function of the length of the connection
    - The greater the connection length, the less the impact of shear lag

Effective Net Area

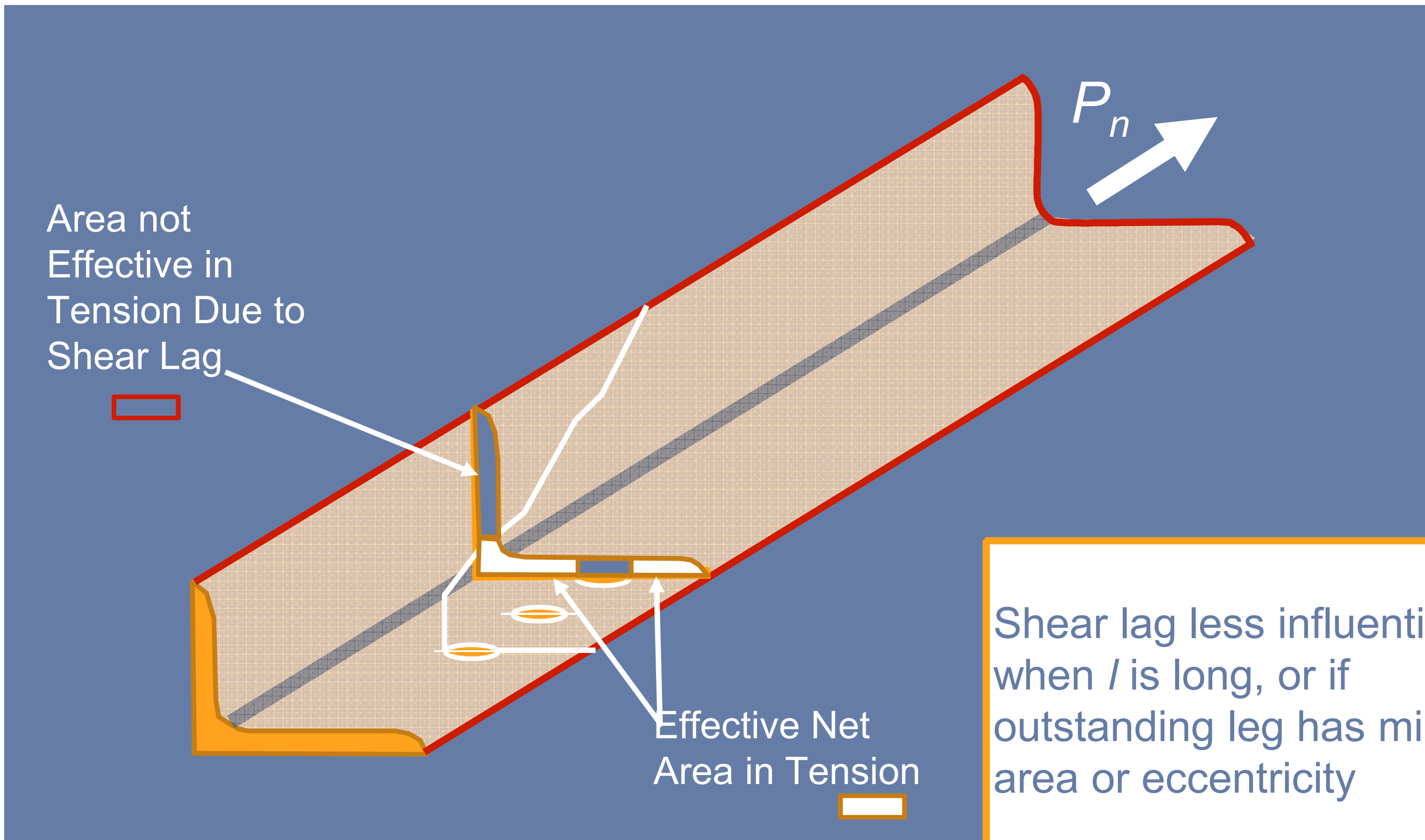
- $A_e = A_n$ when the load is transmitted to each cross-sectional element by connectors.
- $A_e = UA_n$ when the load is transmitted by bolts through some but not all of the cross-sectional elements.
- $A_e = UA_g$ when the load is transmitted by welds through some but not all of the cross-sectional elements.
- U is a reduction factor on the area

Shear Lag



Section Carrying
Tension Forces
P_n
Distribution of
Forces Through
Section
RuptureP
lane
l= Length of Connection

Shear Lag



Shear lag less influential when *l* is long, or if outstanding leg has minimal area or eccentricity

Determining U

- $U = 1 - \frac{\bar{x}}{L}$
  - $\bar{x}$ = distance from the plane of the connection to the centroid of the tension member

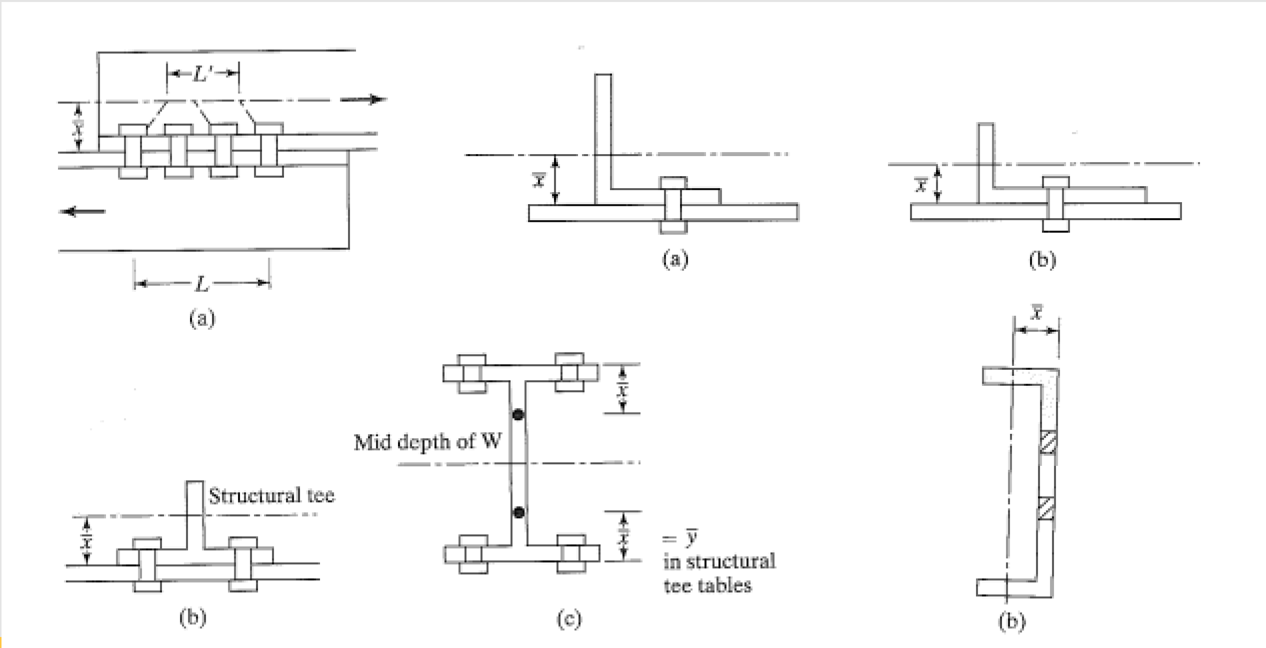
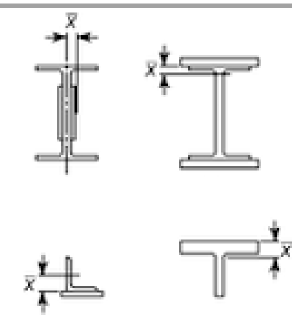
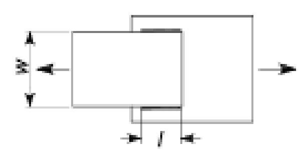
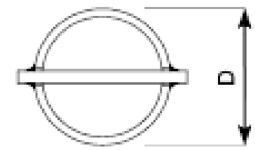
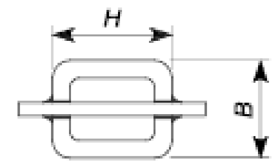
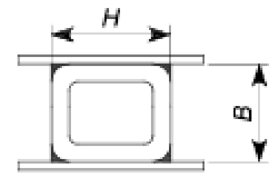


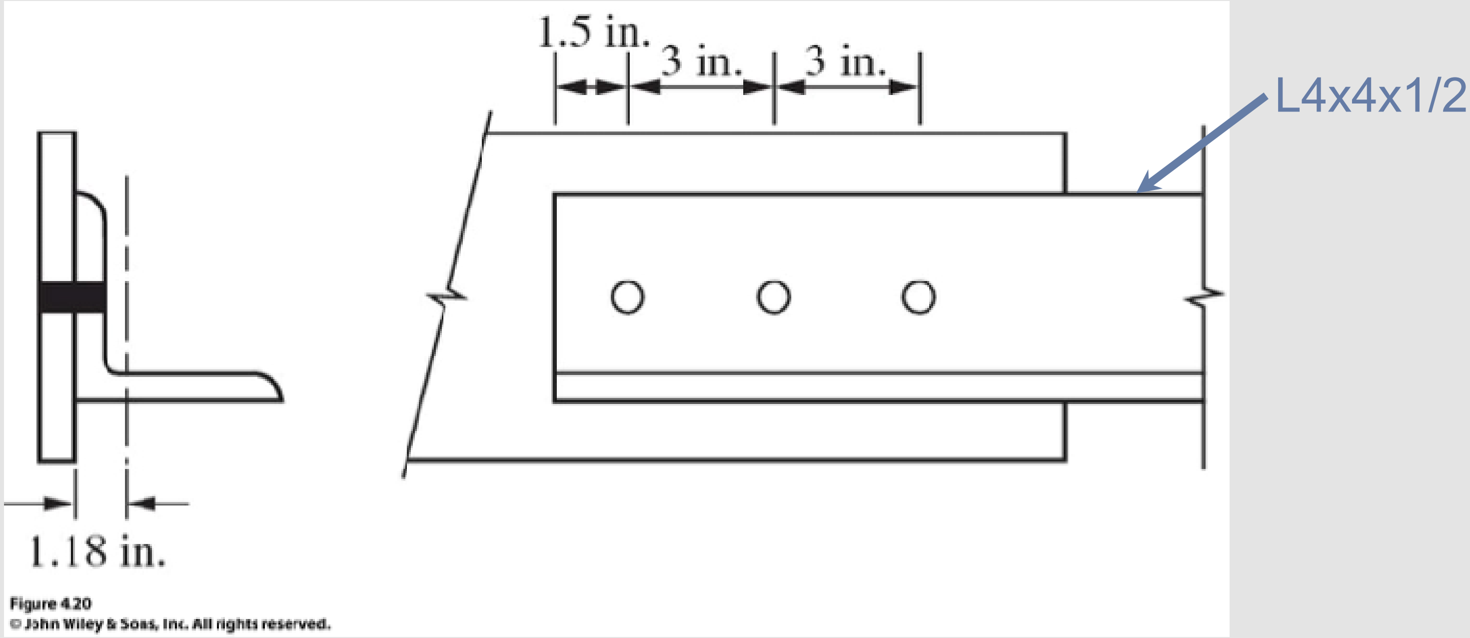
TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element	Shear Lag Factor, *U*	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	———
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds or by longitudinal welds in combination with transverse welds. (Alternatively, for W, M, S and HP, Case 7 may be used. For angles, Case 8 may be used.)	$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and A_n = area of the directly connected elements	———
4	Plates where the tension load is transmitted by longitudinal welds only.	$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm); B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with 3 or more fasteners per line in the direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	—
		with web connected with 4 or more fasteners per line in the direction of loading	$U = 0.70$	—
8	Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with 4 or more fasteners per line in the direction of loading	$U = 0.80$	—
		with 3 fasteners per line in the direction of loading (With fewer than 3 fasteners per line in the direction of loading, use Case 2.)	$U = 0.60$	—

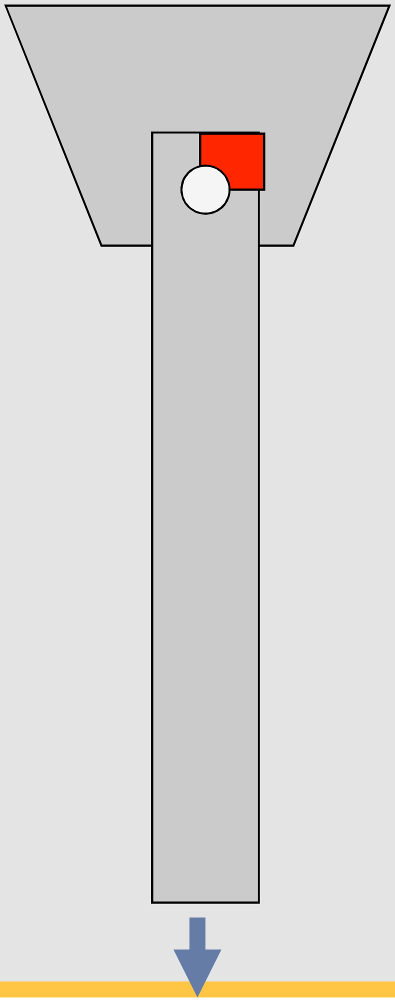
Shear Lag Example



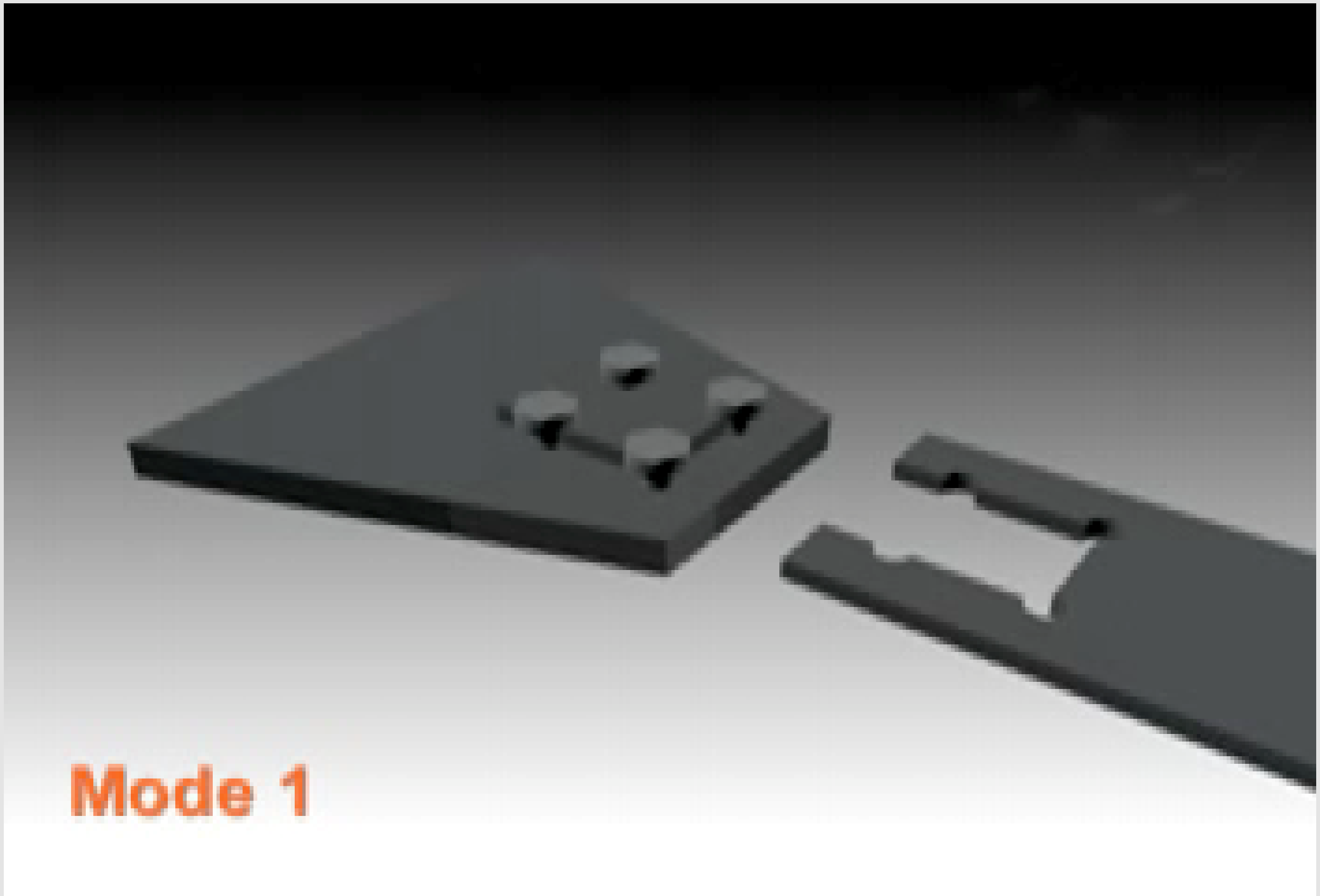
Determine U per Table D3.1

Block Shear

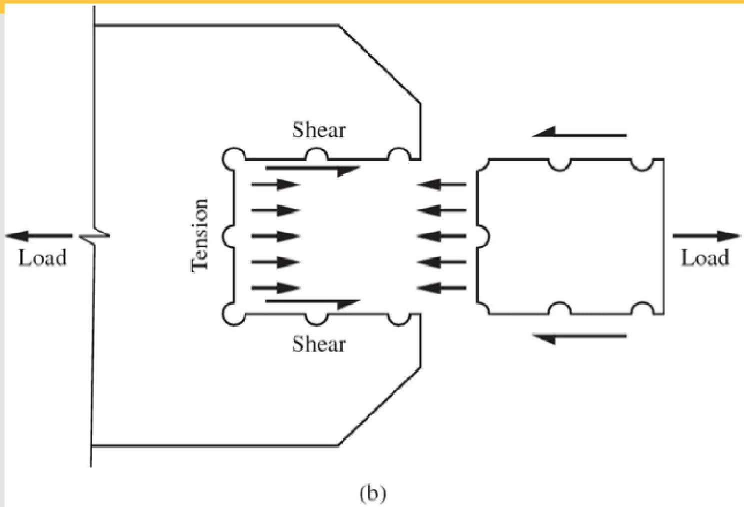
- Tear-out of piece of steel in a connection from combined tensile rupture and shear rupture or shear yield failures



Block Shear



Examples of Block Shear



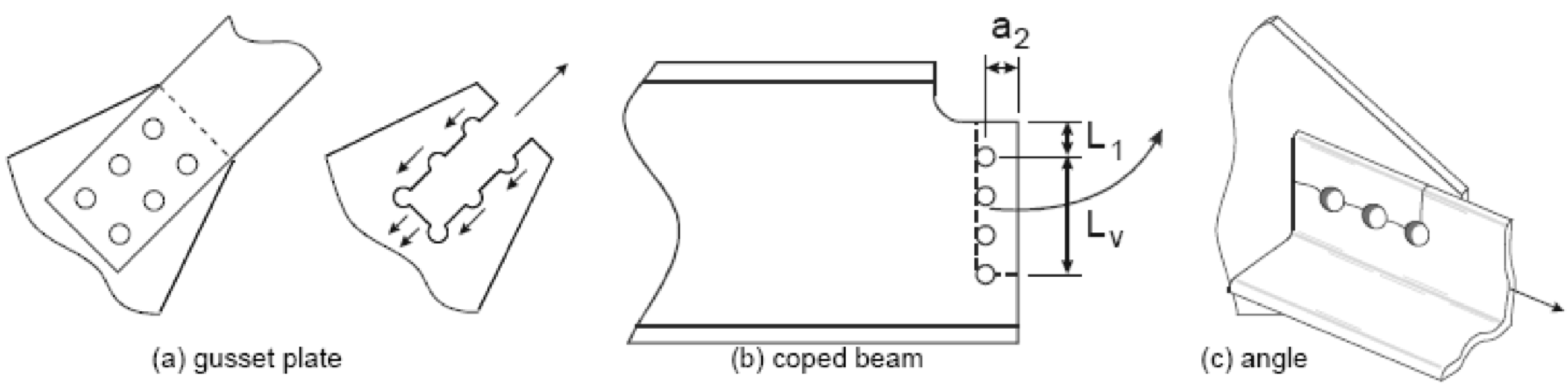
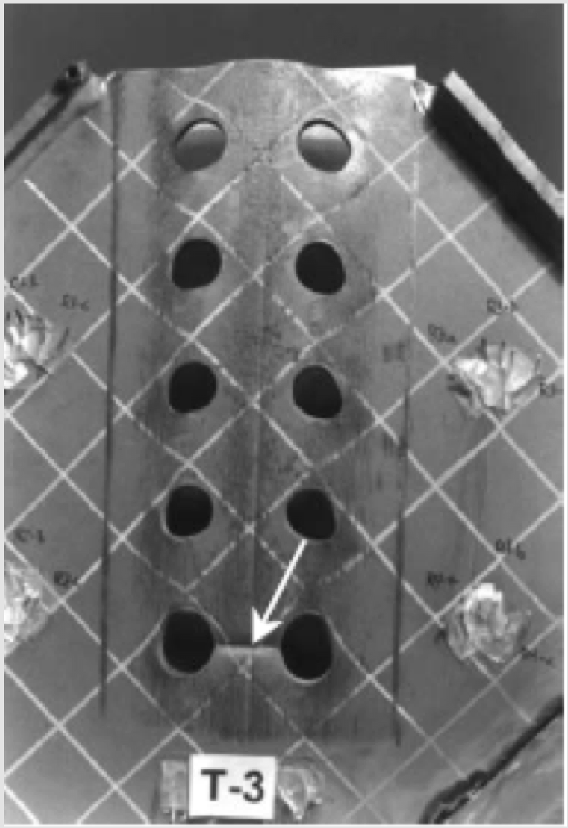


Figure 1 – Examples of Block Shear

Block Shear Failures

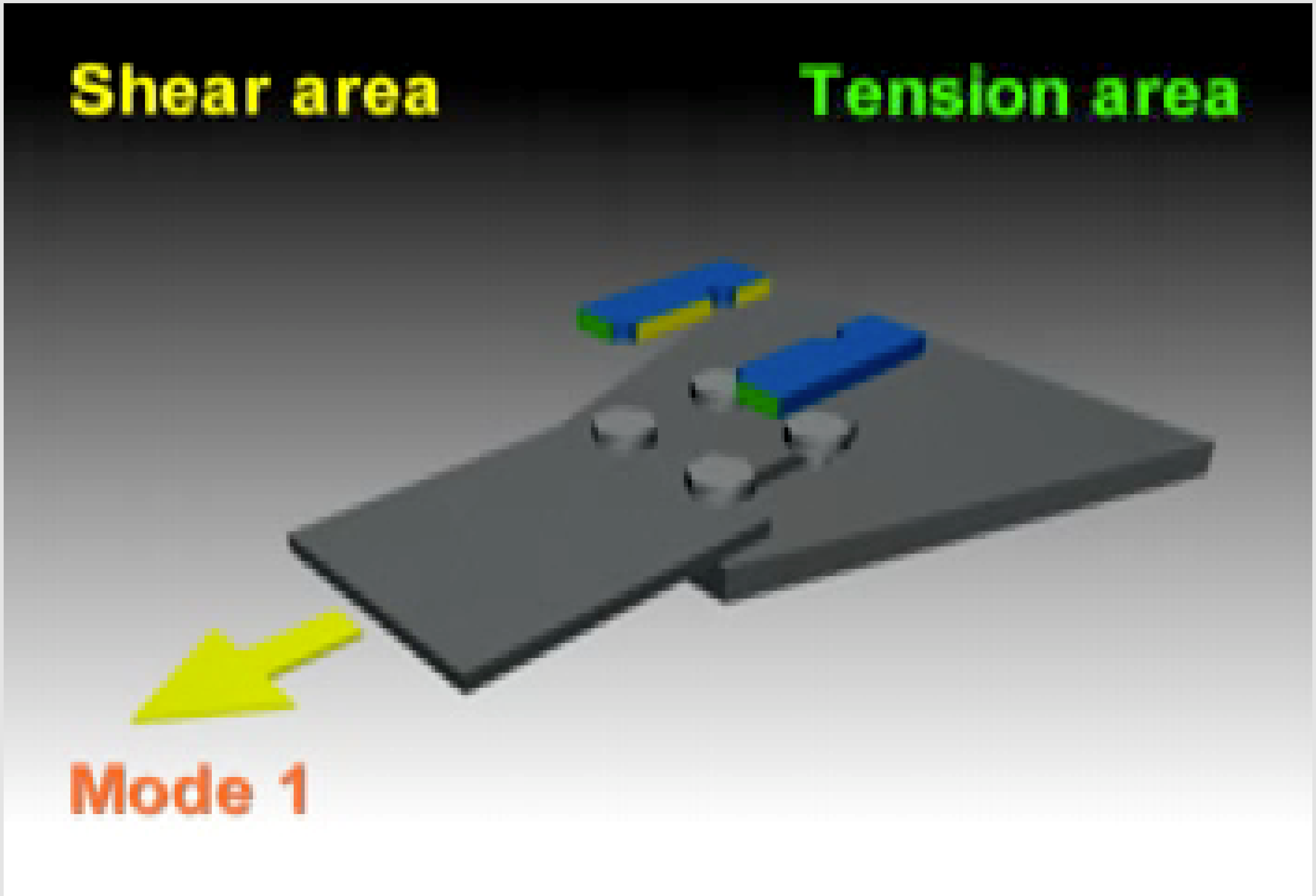
Figure 4.22a
Photo courtesy Robert Driver

Shear Rupture



Shear Yield

Block Shear Areas



Block Shear Components

- Tensile Rupture component

  $F_u\ A_{nt}$

- Shear Rupture & Shear Yield components

  $0.6\ F_u\ A_{nv}$ **(Shear Rupture)**

  $0.6\ F_y\ A_{gv}$ **(Shear Yield)**

Spec J4.3

Block Shear Limit States

$$R_n = 0.6\, F_u\, A_{nv} + U_{bs}\, F_u\, A_{nt}$$
$$\leq 0.6\, F_y\, A_{gv} + U_{bs}\, F_u\, A_{nt}$$

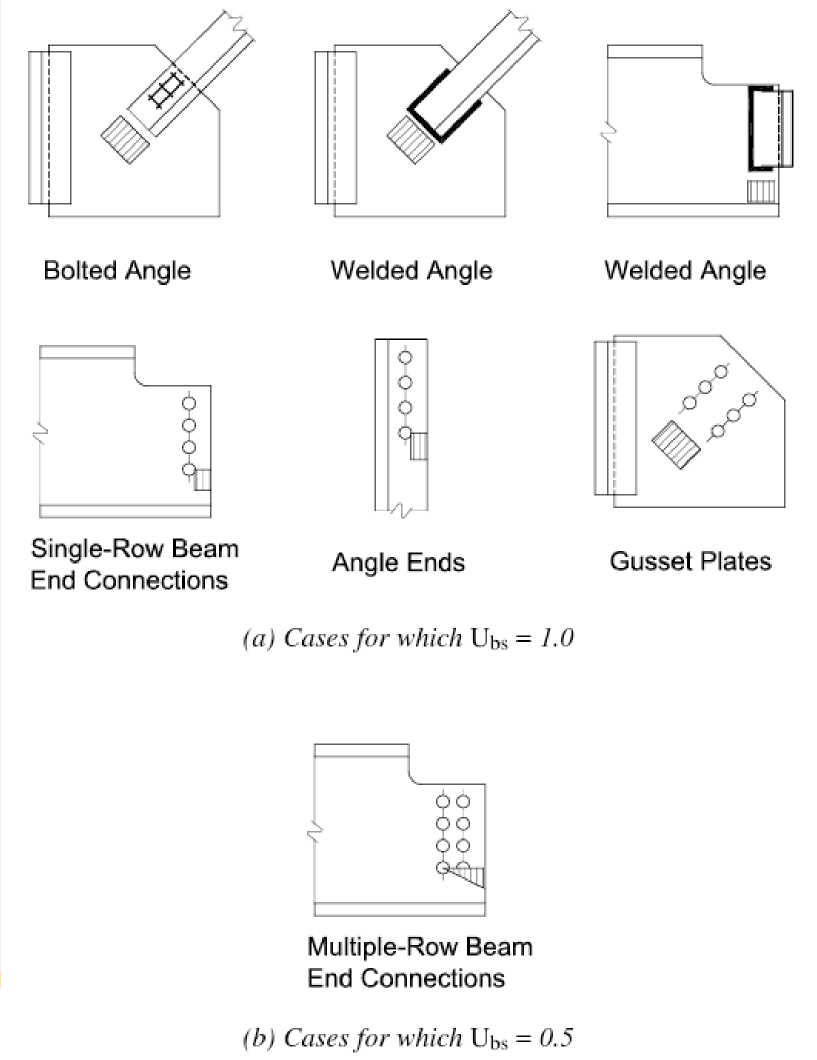
- Tension Rupture included in both cases
- Uses lesser of Shear Rupture or Shear Yield
- For design strength, $\phi = 0.75$

Spec J4.3

Block Shear Reduction Factor

- Uniform Tensile Stress: $U_{bs} = 1.0$
- Non-Uniform Tensile Stress: $U_{bs} = 0.5$
- $U_{bs} = 1.0$ typically applies for tension member connections

Spec C-J4.3



Serviceability Limit State

Serviceability

- Although stability does not affect tension member strength, there is a maximum slenderness ratio suggested.
- Prevents excessive sag and flexibility.
- Preferably $L/r \leq 300$
  - r = radius of gyration = $\sqrt{\frac{I}{A}}$
- Refer to Spec, Section D1

Design Process

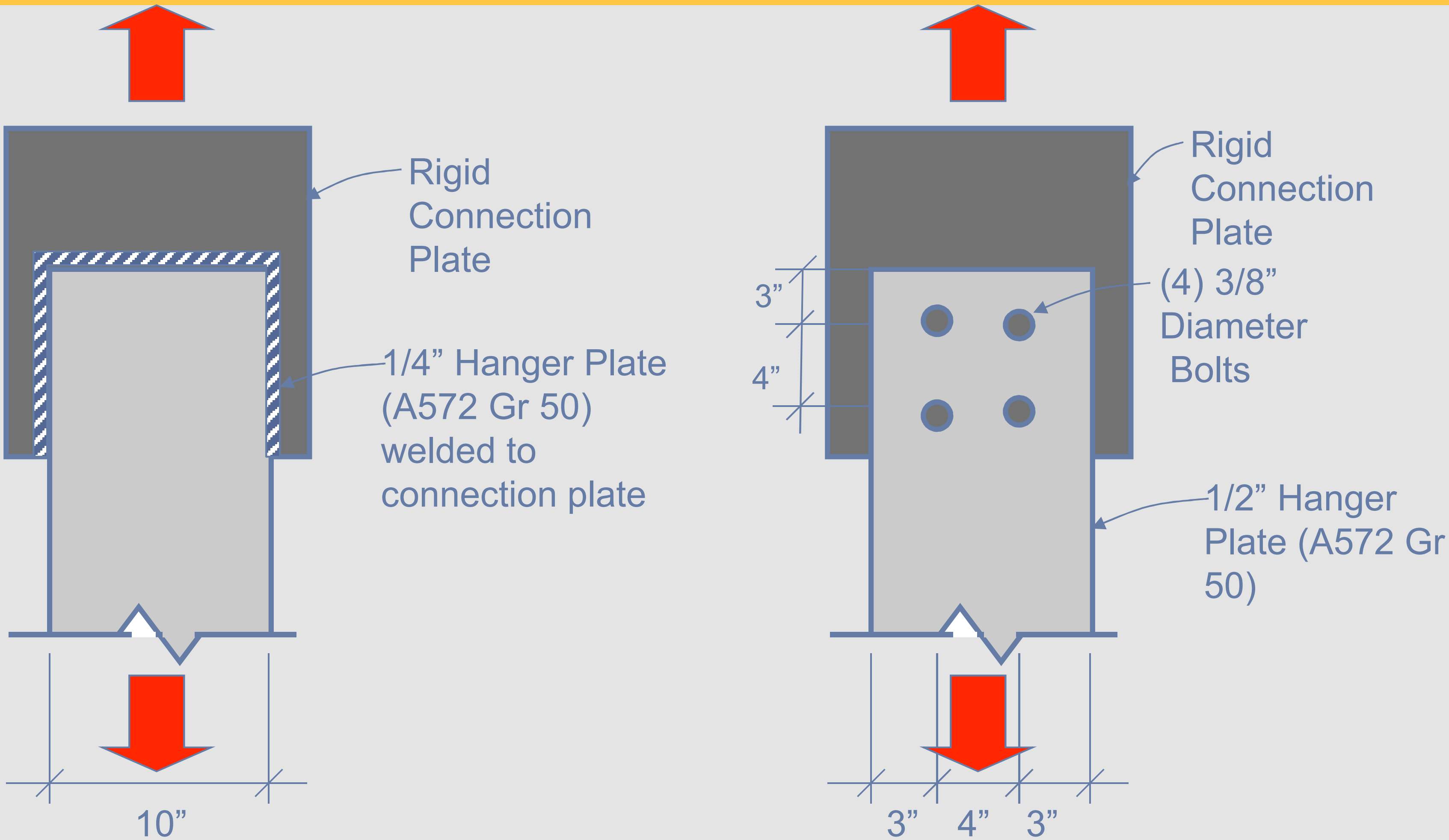
- Required Strength ≤ Design Strength
  - Check all three potential strength limit states
- $P_u \leq$ Lesser of:
  - $0.9\ F_y A_g$
  - $0.75\ F_u A_e$
  - Block Shear strength
- Check Serviceability

Questions?

Example Problem

- You have designed a welded connection detail for the hanger plate shown in the figure labeled "Designed Detail".
- You utilized a welded connection in order to develop the full tensile capacity of the plate.
- The contractor doesn't want to weld the connection so he proposed a bolted connection shown in the figure labeled "Proposed Detail".
- The contractor is aware that the bolts will decrease the capacity of the plate hanger so he proposes thickening the hanger plate as indicated.
- Will you approve of the bolted connection in lieu of the welded connection? Provide all necessary calculations to justify your response.

Example Problem



Designed Detail

Proposed Detail

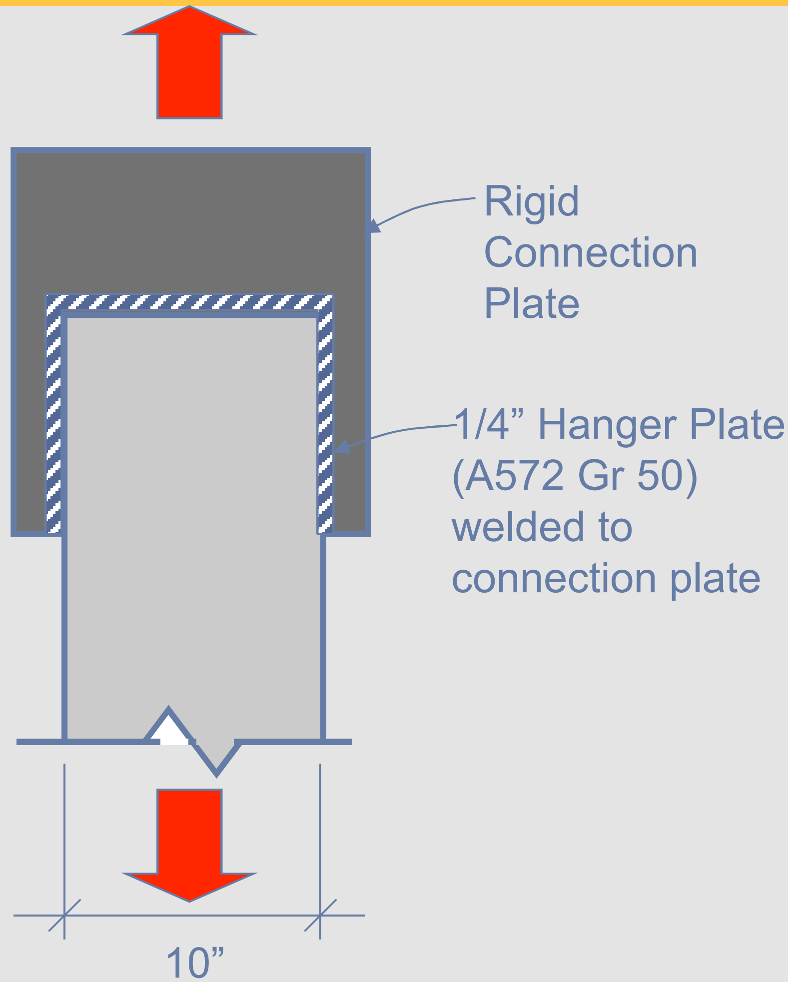
Example Problem

A572 Grade 50 Steel
Fy = 50 ksi
Fu = 65 ksi

Strength of Original Plate:

yielding: $A_g = \frac{1}{4}" \cdot 10" = 2.5\ in^2$

$$\Phi P_n = \Phi F_y \cdot A_g$$
$$= 0.9 \cdot 50 \text{ ksi} \cdot 2.5 \text{ in}^2$$
$$= \mathbf{113\ kips}$$



Designed Detail

Example Problem

A572 Grade 50 Steel
Fy = 50 ksi
Fu = 65 ksi

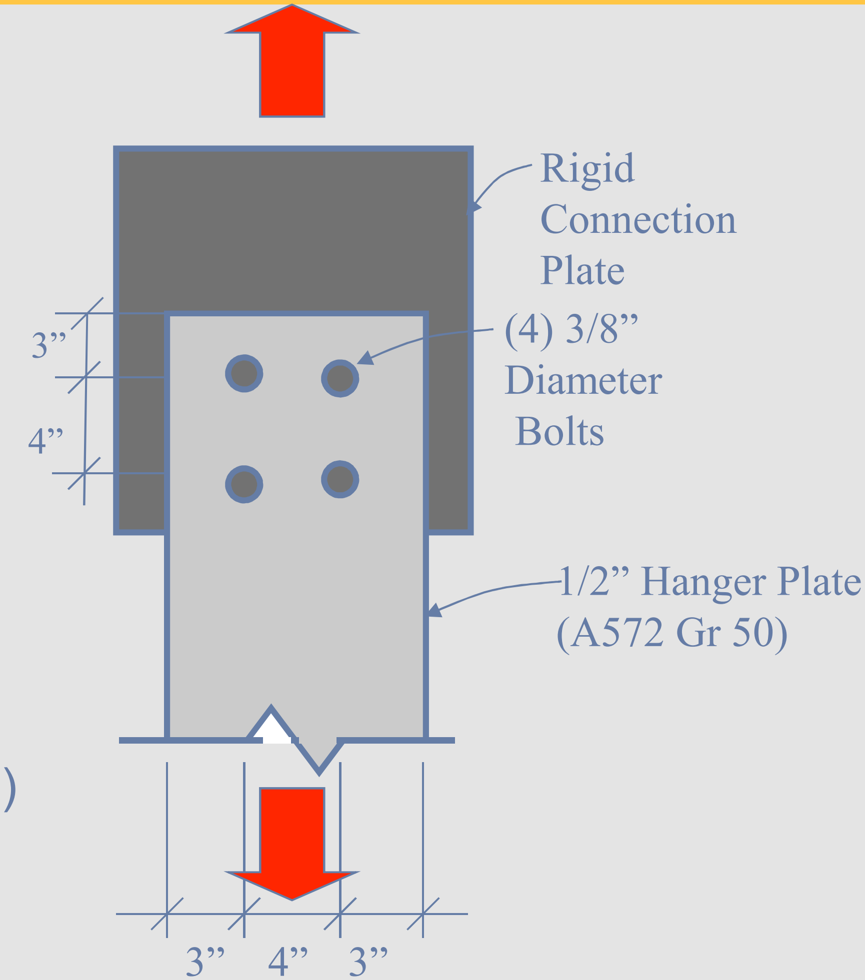
Strength of Proposed Plate:

1. yielding: $A_g = \frac{1}{2}" \cdot 10" = 5.0\ in^2$

$$\Phi P_n = \Phi F_y \cdot A_g$$
$$= 0.9 \cdot 50\text{ ksi} \cdot 5.0\text{ in}^2$$
$$= 225\ kips$$

2. fracture: $An = (\frac{1}{2}")(10" - 2 \cdot ({}^{3}/_{8}" + {}^{1}/_{8}"))$

$$= 4.5\ in^2$$
$$\Phi P_n = \Phi F_u \cdot A_n$$
$$= 0.75 \cdot 65\text{ ksi} \cdot 4.5\text{ in}^2$$
$$= 219\ kips$$



Proposed Detail

Example Problem

A572 Grade 50 Steel

Fy = 50 ksi

Fu = 65 ksi

Nominal Capacity:

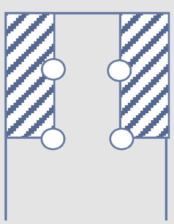
$R_n = 0.6\ F_u\ A_{nv} + U_{bs}\ F_u\ A_{nt}$

$= 0.6\ F_y\ A_{gv} + U_{bs}\ F_u\ A_{nt}$

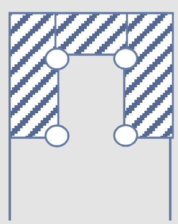
- Use the lesser of the two equations

Strength of Proposed Plate:

3. Block shear:

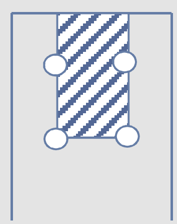


A_{nt} = 2.75 in2
A_{nv} = 6.25 in2
A_{gv} = 7.0 in2
R_n = 423 kips
= 389 kips
ΦR_n = 292 kips



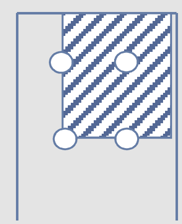
A_{nt} = 4.5 in2
A_{nv} = 3.5 in2
A_{gv} = 4.0 in2
R_n = 429 kips
= 413 kips
ΦR_n = 309 kips

GOVERNING CASE



A_{nt} = 1.75 in2
A_{nv} = 6.25 in2
A_{gv} = 7.0 in2
R_n = 358 kips
= 324 kips
ΦR_n = 243 kips

NOT A REAL CASE



A_{nt} = 3.0 in^2
A_{nv} = 3.0 in2
A_{gv} = 3.5 in2
R_n = 312 kips
= 300 kips
ΦR_n = 225 kips

Example Problem

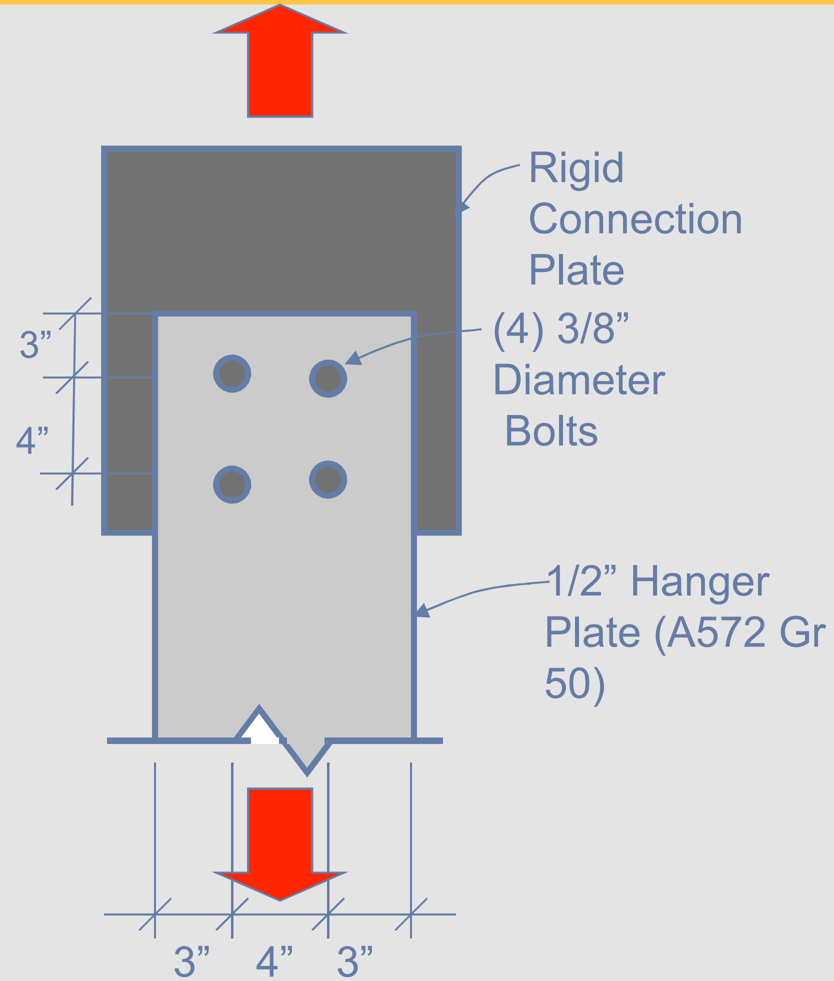
A572 Grade 50 Steel
Fy = 50 ksi
Fu = 65 ksi

Strength of Proposed Plate:

1. yielding: ΦPn = 225 kips

2. fracture: ΦPn = **219 kips**

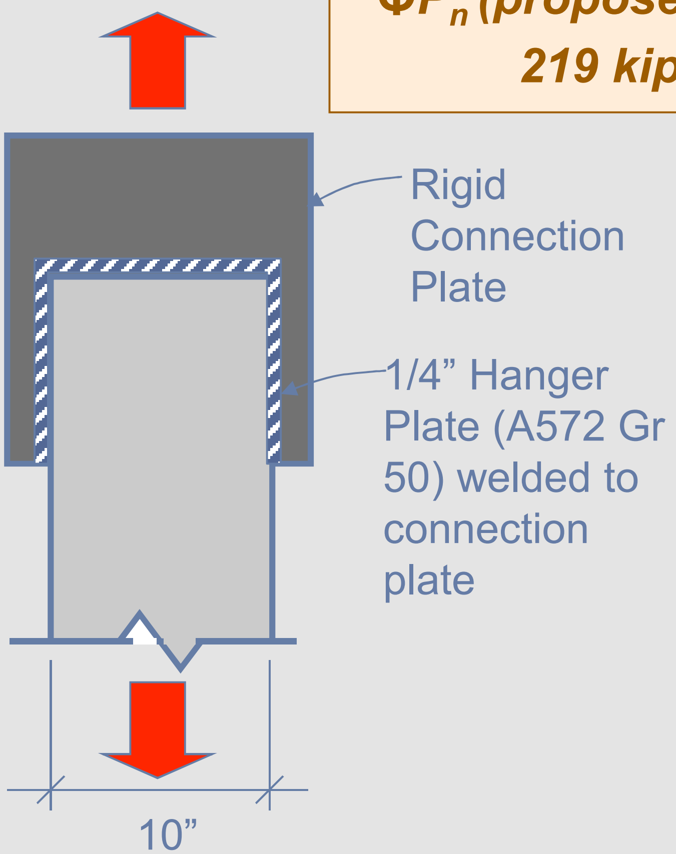
3. Block shear: ΦPn = 243 kips



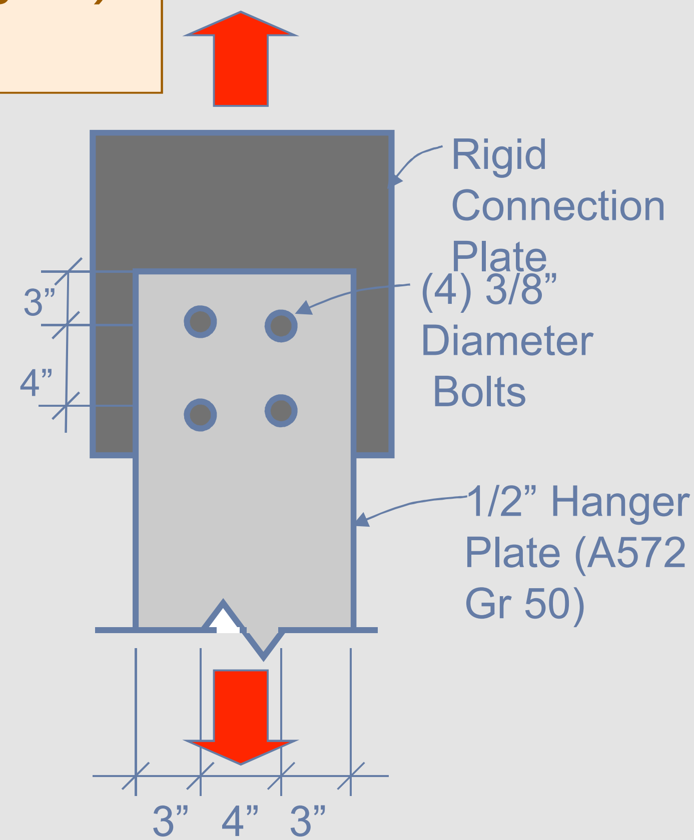
Proposed Detail

Example Problem

CONNECTION APPROVED SINCE
ΦP_n (proposed) > ΦP_n (original)
219 kips > 113 kips



Designed Detail



Proposed Detail

Questions?